O. H. HINSDALE

WAVE RESEARCH LABORATORY

OREGON STATE UNIVERSITY

WRL: Design and Implementation of a Physical Model for Keystone Harbor, WA.

Summary:
The O.H. Hinsdale Wave Research Laboratory (WRL) designed and constructed a 1:40 scale model of Keystone Harbor, WA for Coast & Harbor Engineering (CHE). Upon completion, a series of multi-phase tests we conducted to assess the performance of existing and proposed jetty extensions. The proposed jetty extensions were subjected to long shore currents produced by a pumping system and manifold setup, as well as, waves created using a fully programmable multi-panel piston-type wavemaker. Wave forcing was normally incident and at an oblique angle of -15 degrees from the wavemaker face. These angles corresponded with 165° true N and 180° true N in site coordinates. The testing results for both currents and waves were provided to CHE in tabular form for further analysis.

Preliminary Model Design:

For initial model design considerations, a number of scaling factors were explored to maximize model size while minimizing adverse laboratory effects. Froude similitude was used for scaling because free-surface flows are the dominant process in the model. Froude scaling is applicable for processes in which inertial forces are balanced primarily by gravitational forces, as is the case in most gravity wave problems (Seabergh and Smith 2002). Using Froude similitude, length dimensions are scaled directly with the scaling factor, while time dimensions are scaled as the square root of the scaling factor. Table 1 shows a summary of scaling analysis for three scaling factor options.

Parameter	Prototype		1 : 30		1 : 40		1 : 50	
Depth, D	25	ft	0.83	ft	0.63	ft	0.50	ft
Height, H_min	3	ft	0.10	ft	0.08	ft	0.06	ft
Height, H_max	4	ft	0.13	ft	0.10	ft	0.08	ft
Period, T_min	5	s	0.91	s	0.79	s	0.71	s
Period, T_max	6	s	1.10	s	0.95	s	0.85	s
Jetty Length	312	ft	10.40	ft	7.8	ft	6.24	ft
Jetty Entrance Width	200	ft	6.67	ft	5	ft	4	ft
Peak current	1.2	m/s	0.219	m/s	0.190	m/s	0.17	m/s
Cross sectional area	6,768	m^2	7.53	m^2	4.23	m^2	2.71	m^2
Volumetric flow	$1.3*10^8$	GPM	26,100	GPM	12,700	GPM	7,300	GPM

Table 1. Scaling analysis for scaling factors of 30, 40, and 50.

To determine the scaling of the Keystone Harbor model that would fit the best in the basin and be the most accurate representation of the problem, an area of interest needed to be established. This area included the existing jetty, the area of problematic currents, and the surrounding beach. The area of interest extended out to the 25 meter contour, the depth at which the incoming waves would not be significantly affected. Volumetric flow rates were calculated for the various scaling factors and it was determined that a 1:40 scale was most reasonable. The 1:40 scale still allowed a sufficient area of interest to be examined and required a volumetric flow rate that was achievable with commercial pumps; therefore it was determined to be the best scaling factor for the model.

The physical model of Keystone Harbor, WA was constructed in the tsunami wave basin (TWB) at the WRL. Basin dimensions are as follows: Length 48.8m, Width 27.8m, and Depth 2.1m. The model orientation within the wave basin was aligned so that waves incident from the wavemaker face would correspond with a wave heading of 165° true N as observed at the site. This wave heading was chosen as the mean of dominant wave directions as stated in the CH2M-Hill Feasibility Report (Lilly 2002). Figure 1 shows a layout of Keystone Harbor in the TWB.

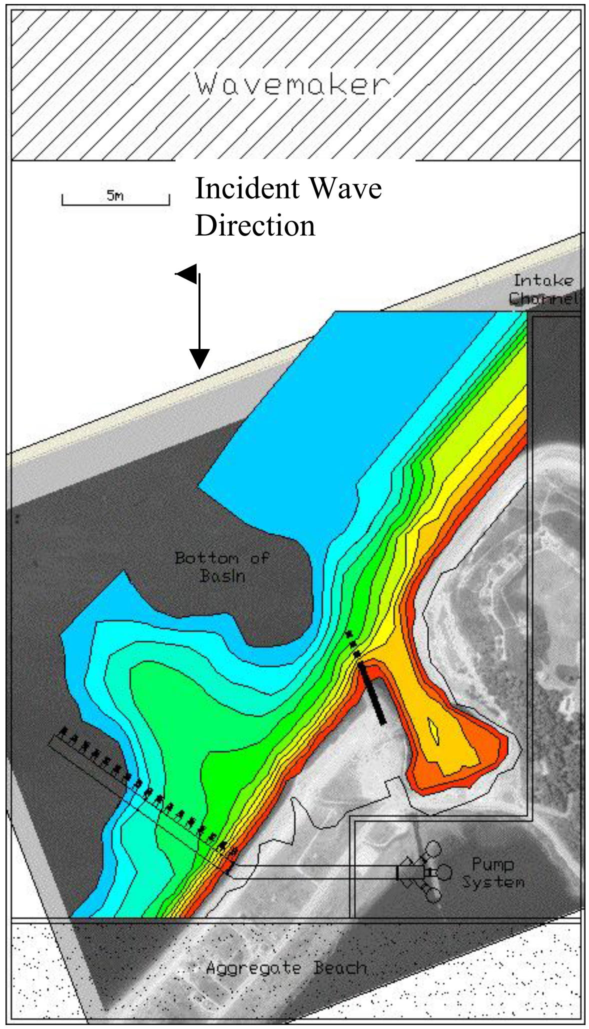


Figure 1. Keystone Harbor 1:40 scale model setup in TWB.

In addition to the design of the Keystone Harbor model, a design for a current generating system was also necessary. Two pump systems were analyzed for effectiveness with the model. An open channel flow was considered that would have an open channel inlet and discharge, while a recirculating pump would be used to supply flow to the system. The advantages of this setup were the ease of installation and having everything at atmospheric pressure. The major disadvantage of this system is the lack of flow control, leading to difficulty in establishing uniformity in the flow field. The second system would utilize a pump and discharge manifold setup. This system would consist of an open channel inlet supplying water to a pump. Attached to the pump is a discharge manifold with valve controls to be used as an outlet for the charged flow. The disadvantages of this system were the expense of the piping and manifold system and head losses within the system. However, this setup had the distinct advantage of flow control from the valves located on the discharge manifold. A similar model study of Ventura Harbor, CA had success using a similar system (Hughes 1998). The pump and discharge manifold setup was chosen since flow control was critical for the study.

Model Construction

I. Model Bathymetry Construction

Prior to the construction of the model bathymetry, a concrete block retaining wall was placed in the basin to provide an intake channel for the pumping system and to separate the existing aggregate beach from the model. Along the intake channel, the wall was built four blocks in height (0.81 m) and one block in width (0.4 m), while the aggregate retaining wall was built to three blocks in height (0.61m) and one block in width. The block wall height difference allowed for realistic land topography to be built along the intake channel.

To provide the greatest accuracy in the construction of the model, a series of forms were needed to establish control points. These transects were oriented shore-normal and were spaced on 2m increments along the beach face. The transects were drawn in an AutoCAD file and the intersection points of the transect lines and bathymetry contours were recorded and plotted for each transect (Figure 2a and 2b).

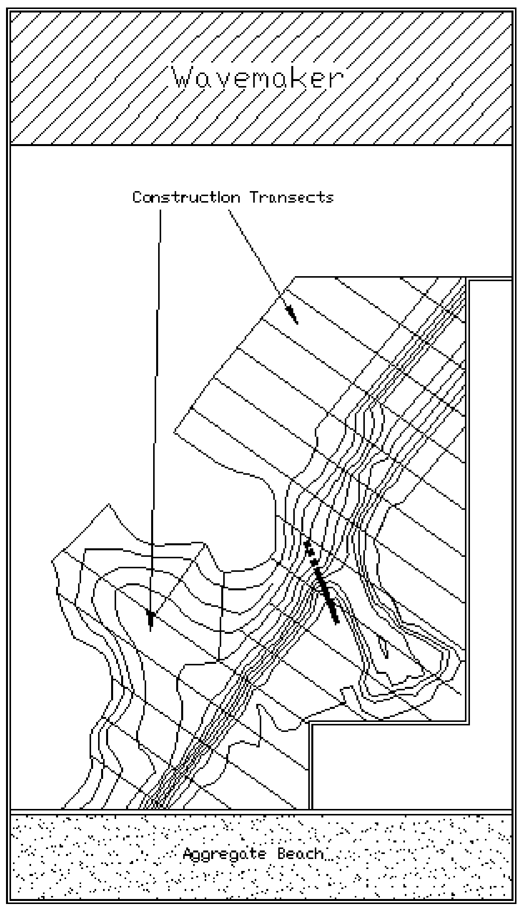


Figure 2a. Construction transect forms.

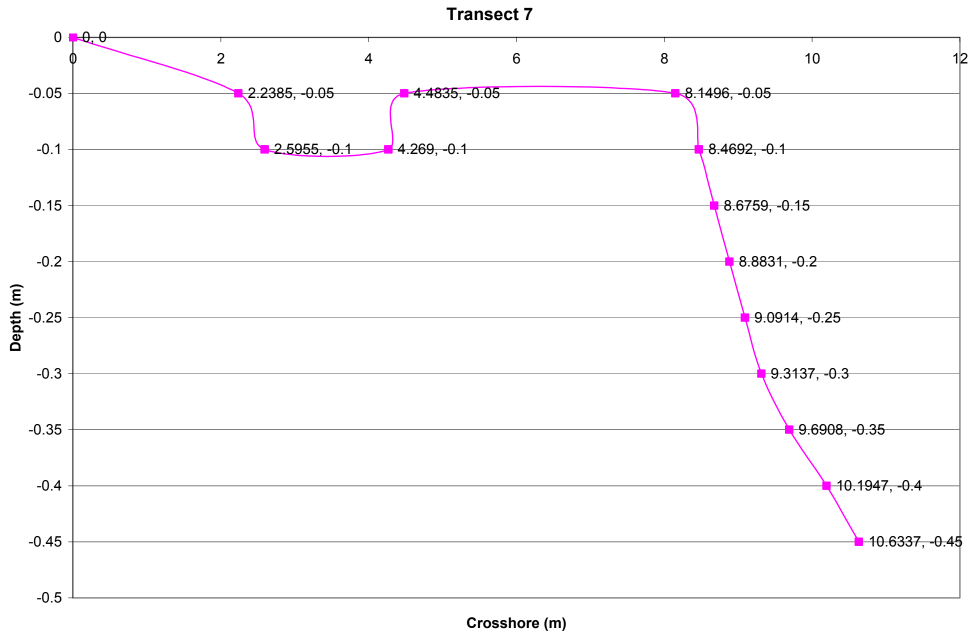


Figure 2b. Transect 7 form layout.

The design of the transect forms with 2 m spacing, allowed construction equipment to be driven between adjacent forms for the placement of sand. Marine grade plywood was cut to the shape of each transect and was then surveyed into its appropriate position within the basin. Figure 3 shows the placement of a transect during construction.

Figure 3. Placement of a plywood transect form.

For stability of the form system, 2m long 2" x 4" studs were attached between transects. Transects were then anchored to uni-strut in the basin floor through the use of angled iron brackets. This anchoring provided a solid base for the form system and alleviated shifting of the model structure during sand placement.

After erection of the transect forms, 200 tons of construction grade sand was placed between transects. Sand placement was done through the use of a Bobcat 763 skid steer equipped with a 0.6 m^3 loading bucket. Additionally, the sand was moved into place by shoveling and raking, then compacted using a jumping jack and plate compactor. For final preparation, the sand was screeded and leveled between transects by dragging a 2" x 4" board. Screeding resulted in a 0.038 m (1 ½ in) space between the sand

and the top of each transect form. This allowed room for placement of a concrete cap which would be flush with the top of each form. The area within the harbor was not contoured by forms, but was appropriately hand shaped based on known contour data.

The final stage of construction required the placement of a 0.038 m (1 ½ in) concrete cap on top of the existing sand bathymetry. The addition of a concrete layer would maintain the correct elevation and bathymetry of the model while protecting it from erosion by waves, currents, and foot traffic. Double Eagle Construction, a local concrete contractor, was employed to consult and assist the WRL on the placement and finishing of concrete. A concrete pumping truck was used for the placement of 14 m^3 of the wet concrete and finishing was undertaken by project staff. Pumping, screeding, and finishing of the concrete took four hours with the help of two experienced finishers, and nine other less experienced workers. A 1 m bull float with a 5 m long handle was used to create a smooth finish between each transect. Figure 4 shows the finishing of concrete after placement.

Figure 4. Concrete finishing using a 1 m bull float.

The green concrete was allowed to cure for a period of 72 hours before additional finishing was undertaken. After this period, rough edges of the concrete were ground down using a 0.18 m (7 in) diameter wheel grinder, and the model and basin were pressure washed to prepare for final grout application. A thin set grout was applied to all of the cracks and crevices present on the model. This created smooth transitions along the model surface and minimized any obstructions that might have adverse effects to the water flow.

II. Pump and Manifold System

Initial design for the current generating system required a volumetric flow of 12,700 GPM. This flowrate could be achieved by attaching a series of three submersible pumps rated at 5,000 GPM apiece, to an intake manifold. Figure 5 shows a schematic of the pump and discharge manifold system designed for generation of currents. The orientation of the pump and manifold system relative to the basin can be seen in Figure 1.

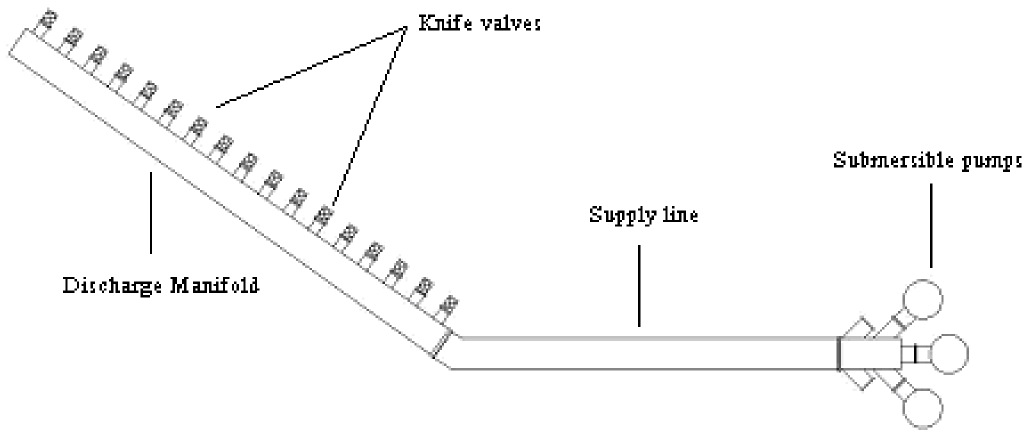


Figure 5. Pump and discharge manifold setup for generation of currents in TWB.

The supply line and discharge manifold were constructed of 0.6 m (24 in) diameter high density polyethylene (HDPE) pipe. The supply line was built to a length of 7.5 m and fitted with flanged adapters at each end. One end connected to the submersible pump intake manifold, while the other was connected to the discharge manifold. The discharge manifold was capped on one end and fitted with a flanged adapter on the other. The discharge manifold also consisted of seventeen, 0.2 m (8 in.) diameter knife valves mounted normally to the main water line. Figure 6 shows the discharge manifold and knife valves. At the far end of the discharge manifold, a 1.2 m x 2.4 m sheet of galvanized sheetmetal was placed to provide a smooth flow transition from the basin floor to model contours.

Figure 6. Current discharge manifold and knife valves.

Currents were generated with BIBO BS-2250 industrial dewatering pumps, rated at 5000 GPM each. Flow skirts were placed around each pump to minimize vortexing and water drawdown at the pump intake. Knife valves were used to adjust flow volumes and to create the desired flow field. This process will be discussed in greater detail in the next section.

III. Jetty Construction

Model jetty footprints were surveyed onto the model topography and marked off with waterproof crayon. All riprap jetties were constructed of 54.5 mm aggregate. Berm width and lengths were verified after placement. All slopes and lengths were in accordance with plans provided by CHE.

Alternative 4A, the pile supported wave barrier, was constructed of fifteen, 2.2 cm O.D. copper pipe piles. These piles were placed at 0.3 m intervals along the wave barrier. The wave barrier extended to 2/3 the water depth and was constructed of two sheets of marine grade plywood. Total width of wave barrier was 0.6 m.

Experimental Setup and Data Collection:

I. Currents

Initial calibration of the longshore currents for testing resulted in two major changes:

1. Installation of current guide wall at the end of the discharge manifold. The guidewall minimized gyres and eddies occurring due to viscous effects between still and moving water. Preliminary observation and analysis proved wall effects to be minor. A similar method was used by Simons et. al. (1995).
2. Installation of only two, BIBO BS-2250 submersible pumps. Initial findings showed that required flowrates could be achieved with one less pump than design calculations suggested.

Calibration of the model was undertaken with no jetty in place. An iterative process of adjusting knife valves and obtain current velocity measurements was used to achieve the final flow field for the testing of all jetty alternatives. To determine the length of time between starting the pump and the reaching of steady state, a calibration test was run for a duration of 2 hours. Analysis of this data determined that a steady state velocity in the basin occurred after approximately 5 minutes. Further analysis of the steady state velocity showed that a 2% difference in recorded current velocities existed between a 3-minute and 10-minute averaging period. Therefore it was decided that 3-minute averaging would be used while collecting current velocity measurements.
A data sampling rate of 1 Hz was used during data collection.

Current velocity measurements were obtained using an array of four, 3-D Sontek Acoustic Doppler Velocimeters (ADV's). These ADV's were mounted on an instrument frame on one meter intervals. The instrument frame was attached to a movable bridge, spanning the entire width of the TWB. This setup allowed a representative cross section of the velocity field to be sampled for analysis. The current velocity measurement grid is shown in Figure 7.

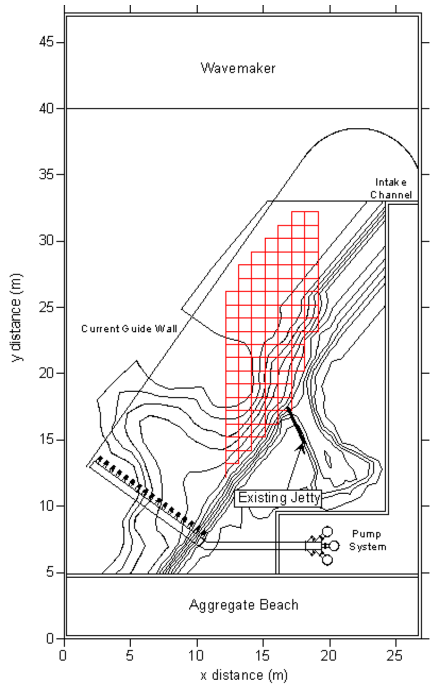


Figure 7. Current velocity measurement grid. Courtesy of CHE.

Figure 8 shows a picture of the ADV measurement setup.


Figure 8. ADV instrument frame used for current velocity measurements.

Experimental Results: Currents

The following plots summarize the current vector fields for each jetty design alternative.

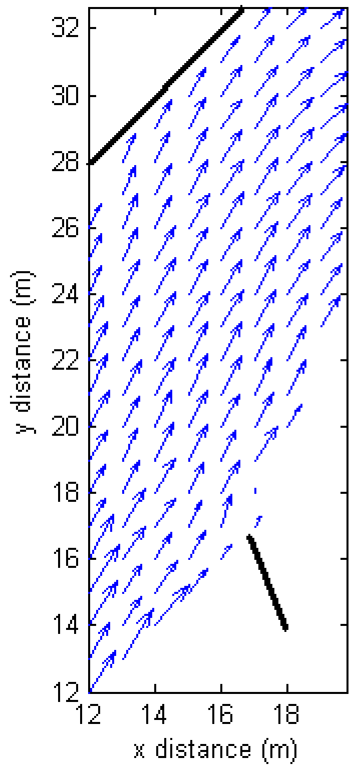


Figure 9. Current velocity vector field, Existing Conditions

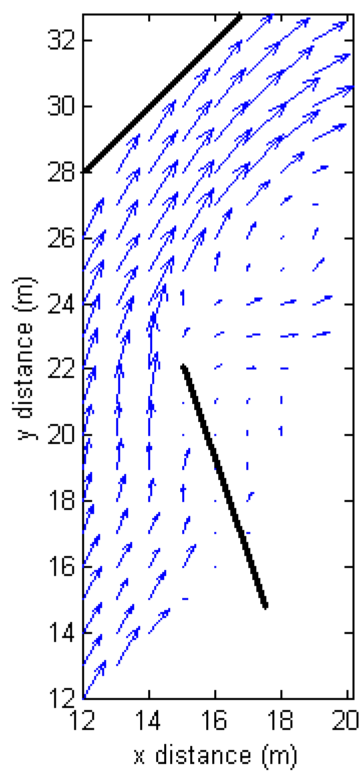


Figure 10. Current velocity vector field, ALT-1A

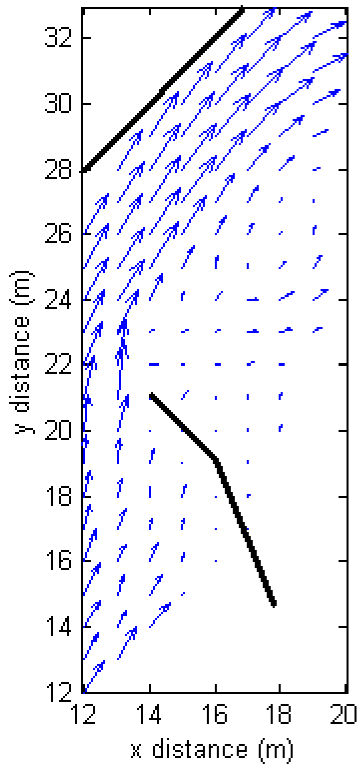
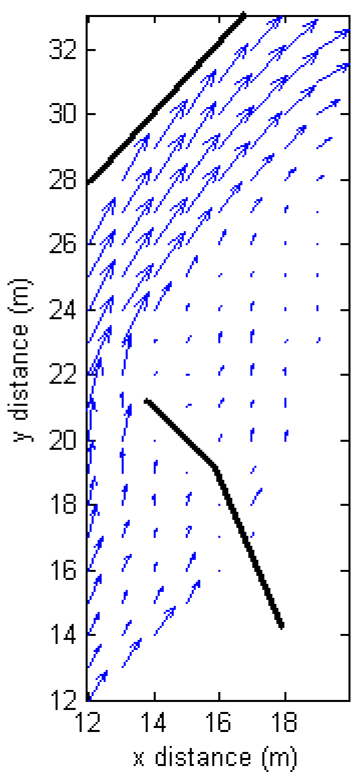


Figure 11. Current velocity vector field, ALT-



3A

Figure 12. Current velocity vector field, ALT-3B

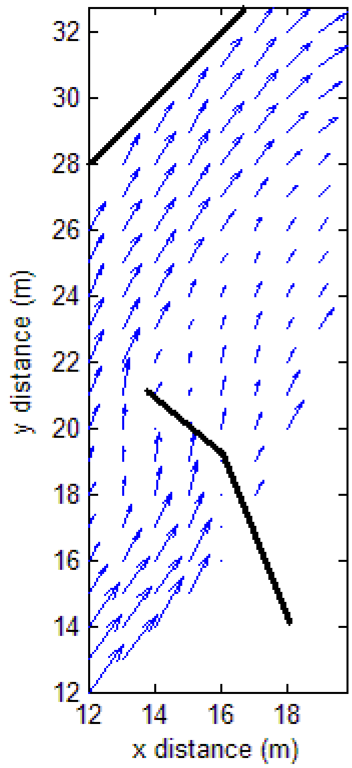


Figure 13. Current velocity vector field, ALT-4A

Tabular data of the current velocity vector fields have been provided to CHE for additional analysis.

II. Waves

Prior to wave testing, the current guide wall was removed so as not to impede the incident wave field. Additionally, a synthetic matting of wave absorbing material was hung along the discharge manifold and exposed basin wall. This process minimized wave reflection off of these objects during testing.

A series of irregular and monochromatic wave trains were run at each jetty alternative to qualitatively assess shoreline change near the jetty extension and observe channel diffraction patterns. Each timeseries was run for 20 minutes. Table 2 gives a summary of the characteristics of each wave sequence run.

Case #	Prototype θ,H,T	Wave Type	Model θ,H,T
1	165˚, 3 m, 4 s	Regular	0˚, 2.4 cm ,0.63 s
2	180˚, 3 m, 4 s	Regular	-15˚, 2.4 cm ,0.63 s
3	165˚, 4 m, 5 s	Regular	0˚, 3.0 cm, 0.79 s
4	180˚, 4 m, 5 s	Regular	-15˚, 3.0 cm, 0.79 s
5	165˚, 3 m, 4 s	Random	0˚, 2.4 cm, 0.63 s
6	165˚, 4 m, 5 s	Random	0˚, 3.0 cm, 0.79 s

Table 2. Characteristics of 6 wave cases used in wave testing sequence.

Wave height measurements were obtained using three surface piercing, resistance-type wave gages. A sampling rate of 50 Hz was used during data collection. Wave gages were mounted on the same instrument frame as described above. Figure 13 and Figure 14 show schematics of regular wave and random wave data collection points, respectively.

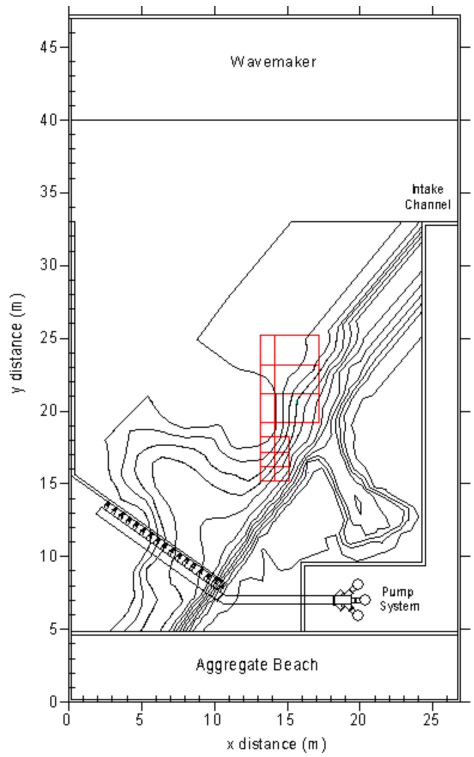


Figure 13. Data collection grid for regular wave data. Courtesy of CHE.

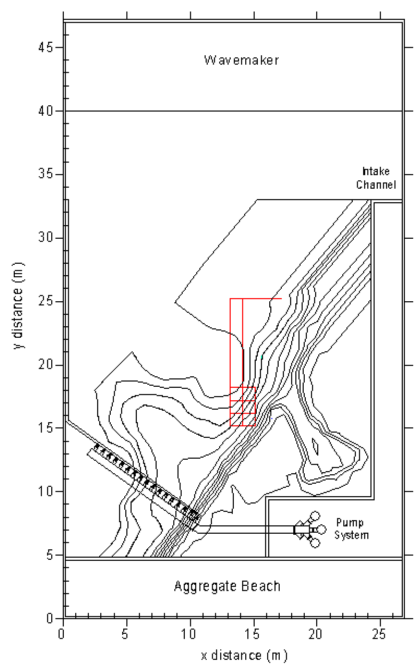


Figure 14. Data collection grid for random wave data. Courtesy of CHE.

For the purpose of shoreline change analysis, a 5 cm (2 in) veneer of scaled cobble and sand material was placed adjacent to the jetty toe. This thin veneer of beach material stretched along the beach face approximately 4 m from the jetty and extended seaward to a water depth of 45 cm. The material was comprised of two sizes of silica sand mixed in a 1:1 ratio. It is important to note that the d_{50} grain sizes of sediments were scaled so that the fall velocity of the prototype sediments matched the scaled fall velocity of the model sediments; they were not scaled via the length scale (Hughes, 1993). Table 3 gives nominal dimensions of silica grain materials used.

Parameter	**Prototype**		**1 : 40**	
d_{50} sand	0.3	mm	0.095	Mm
d_{50} cobble	3.0	cm	1.05	Mm

Table 3. Nominal dimensions of silica sand used for beach materials

Before each wave case was run, the sediment beach adjacent to the jetty was groomed to a smooth and uniform finish. A record of digital photographs was taken throughout the 20 minute testing period. These photographs were taken prior to testing, and at 1,10, and 20 minute intervals. Along with the digital photograph record, detailed notes on shoreline change observations were taken during testing. These notes included observations in diffraction patterns, sediment transport, and berm formations. The digital record and notes have both been provided to CHE.

Experimental Results: Waves

The following tables delineate the results of the wave testing for each jetty alternative.

Keystone_H_Existing

Case	Y-Location	Theta [deg]	WB [cm]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean H1:H3	Target [cm]	Percent Diff [%]
1	incident	0	4.4	2.1	1.9	2.4	2.1	2.3	-7
1	25	0	4.0	2.4	2.1	2.3	2.3	2.3	-1
1	23	0	4.1	2.0	2.2	2.3	2.1	2.3	-7
1	21	0	5.2	3.7	3.2	2.9	3.3	2.3	43
1	19	0	4.6	2.6	2.2	2.1	2.3	2.3	0
1	18	0	4.3	2.7	2.8	2.1	2.5	2.3	10
1	17	0	4.4	2.7	2.9	1.6	2.4	2.3	5
1	16	0	4.6	2.3	3.3	2.1	2.6	2.3	13
1	15	0	4.4	2.2	2.4	2.1	2.3	2.3	-1
2	incident	-15	3.3	3.4	2.5	1.7	2.5	2.3	10
2	25	-15	3.6	4.4	3.2	1.8	3.1	2.3	37
2	23	-15	4.0	3.0	3.8	2.4	3.1	2.3	35
2	21	-15	3.9	2.9	3.6	4.0	3.5	2.3	53
2	19	-15	3.8	1.6	2.9	3.1	2.5	2.3	11
2	18	-15	4.0	3.0	1.8	2.8	2.5	2.3	11
2	17	-15	3.9	2.2	3.1	3.0	2.8	2.3	21
2	16	-15	3.9	3.8	3.0	3.6	3.5	2.3	52
2	15	-15	3.7	3.2	2.6	3.0	3.0	2.3	29
3	incident	0	4.7	2.6	3.0	3.0	2.9	3.0	-5
3	25	0	4.4	3.2	2.8	3.6	3.2	3.0	7
3	23	0	4.0	3.0	3.8	2.4	3.1	3.0	3
3	21	0	4.8	2.7	3.5	2.6	2.9	3.0	-2
3	19	0	5.0	3.4	2.9	3.4	3.2	3.0	6
3	18	0	5.1	3.3	3.3	3.4	3.4	3.0	12
3	17	0	4.9	3.1	3.1	2.5	2.9	3.0	-4
3	16	0	5.4	3.3	3.2	3.2	3.2	3.0	8
3	15	0	5.5	2.9	3.2	2.8	2.9	3.0	-2
4	incident	-15	4.3	3.5	2.5	3.7	3.2	3.0	7
4	25	-15	5.2	3.3	2.8	3.8	3.3	3.0	11
4	23	-15	5.2	3.7	3.2	2.9	3.3	3.0	9
4	21	-15	5.0	3.3	3.0	2.3	2.8	3.0	-5
4	19	-15	4.8	3.8	3.7	3.7	3.7	3.0	24
4	18	-15	4.8	2.5	3.4	2.2	2.7	3.0	-10
4	17	-15	5.1	3.7	3.4	4.4	3.9	3.0	28
4	16	-15	5.4	2.5	2.3	3.1	2.6	3.0	-13
4	15	-15	5.7	3.9	1.7	4.0	3.2	3.0	7
5	incident	0	3.8	2.2	2.1	2.3	2.2	2.3	-5
5	19	0	3.9	2.3	2.2	2.2	2.2	2.3	-2
5	17	0	3.9	2.2	2.2	2.3	2.2	2.3	-3
5	15	0	3.9	2.2	2.1	2.2	2.2	2.3	-5
6	incident	0	4.7	2.9	2.8	2.9	2.9	3.0	-5
6	19	0	4.8	3.0	3.0	3.3	3.1	3.0	3
6	17	0	4.8	3.1	2.8	3.2	3.0	3.0	1
6	15	0	4.8	3.1	2.7	3.1	2.9	3.0	-2

Table 5. Summary of wave field for Existing Condition

Keystone_H_Alt1A									
Case	Y-Location	Theta [deg]	WB [cm]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean H1:H3	Target [cm]	Percent Diff [%]
1	incident	0	3.8	2.2	2.2	2.4	2.3	2.3	-1
1	25	0	3.6	1.9	1.9	1.9	1.9	2.3	-16
1	23	0	3.9	1.8	2.7	2.7	2.3	2.3	-1
1	21	0	3.6	2.5	0.0	2.6	1.7	2.3	-26
1	19	0	4.1	2.2	0.8	1.9	1.6	2.3	-28
1	18	0	4.2	2.3	1.6	1.5	1.8	2.3	-22
1	17	0	4.3	2.8	1.2	1.1	1.7	2.3	-26
1	16	0	4.5	2.7	1.1	1.2	1.7	2.3	-27
1	15	0	4.5	2.6	0.7	1.8	1.7	2.3	-27
2	incident	-15	3.1	3.2	2.5	1.6	2.4	2.3	7
2	25	-15	3.9	4.5	3.7	1.3	3.2	2.3	38
2	23	-15	3.8	2.7	4.0	3.1	3.3	2.3	43
2	21	-15	4.1	2.2	0.0	2.0	1.4	2.3	-39
2	19	-15	4.5	2.6	1.0	1.6	1.8	2.3	-23
2	18	-15	2.9	2.6	0.9	2.0	1.8	2.3	-19
2	17	-15	3.8	2.3	1.1	2.1	1.9	2.3	-19
2	16	-15	4.1	3.2	0.7	1.6	1.8	2.3	-20
2	15	-15	4.5	2.0	0.9	0.8	1.2	2.3	-46
3	incident	0	4.9	2.8	3.0	3.3	3.1	3.0	2
3	25	0	4.5	3.3	3.6	4.1	3.7	3.0	22
3	23	0	4.7	3.3	2.6	3.3	3.0	3.0	2
3	21	0	4.9	3.3	0.0	3.7	2.3	3.0	-23
3	19	0	5.1	3.4	1.2	3.0	2.5	3.0	-16
3	18	0	5.5	3.6	1.1	3.0	2.6	3.0	-15
3	17	0	5.5	3.3	1.3	2.0	2.2	3.0	-27
3	16	0	5.7	3.5	1.8	3.0	2.8	3.0	-8
3	15	0	5.7	3.2	1.3	2.4	2.3	3.0	-24
4	incident	-15	4.2	3.5	2.3	3.6	3.1	3.0	5
4	25	-15	4.7	3.6	2.6	4.1	3.4	3.0	15
4	23	-15	4.8	3.3	3.4	3.0	3.2	3.0	7
4	21	-15	5.1	3.3	0.0	1.7	1.7	3.0	-45
4	19	-15	5.1	2.8	0.5	2.0	1.8	3.0	-41
4	18	-15	5.0	1.2	1.0	1.8	1.3	3.0	-56
4	17	-15	5.3	2.6	0.9	3.4	2.3	3.0	-23
4	16	-15	5.1	1.9	2.2	0.8	1.7	3.0	-45
4	15	-15	5.1	2.3	1.7	1.3	1.8	3.0	-41
5	incident	0	3.8	2.2	2.1	2.3	2.2	2.3	-6
5	19	0	3.9	2.5	0.7	1.9	1.7	2.3	-26
5	17	0	3.9	2.4	1.0	1.7	1.7	2.3	-26
5	15	0	3.9	2.3	1.0	1.6	1.6	2.3	-30
6	incident	0	4.7	2.9	2.7	3.0	2.9	3.0	-4
6	19	0	4.8	3.2	1.1	2.3	2.2	3.0	-27
6	17	0	4.7	2.9	1.3	2.2	2.1	3.0	-29
6	15	0	4.7	2.8	1.3	2.1	2.1	3.0	-31

Table 5. Summary of wave field for Alternative 1A

Keystone_H_Alt3A

Case	Y-Location	Theta [deg]	WB [cm]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean H1:H3	Target [cm]	Percent Diff [%]
1	incident	0	3.8	2.1	2.5	2.1	2.2	2.3	-2
1	25	0	3.8	1.9	2.0	1.3	1.8	2.3	-23
1	23	0	3.7	1.9	2.0	2.2	2.0	2.3	-11
1	21	0	4.0	2.2	2.6	2.4	2.4	2.3	5
1	19	0	4.4	1.9	0.1	0.6	0.9	2.3	-62
1	18	0	4.4	2.1	0.6	0.5	1.1	2.3	-54
1	17	0	4.5	2.4	1.0	1.0	1.5	2.3	-36
1	16	0	4.6	2.1	1.1	1.3	1.5	2.3	-35
1	15	0	4.5	1.6	1.0	1.5	1.4	2.3	-38
2	incident	-15	2.3	1.8	2.8	2.8	2.5	2.3	9
2	25	-15	2.7	1.8	3.1	2.5	2.5	2.3	8
2	23	-15	3.2	3.4	2.2	4.0	3.2	2.3	39
2	21	-15	3.2	4.3	3.0	4.2	3.8	2.3	67
2	19	-15	3.3	3.0	0.3	0.5	1.3	2.3	-44
2	18	-15	3.5	1.4	0.7	0.5	0.9	2.3	-62
2	17	-15	3.2	1.3	0.8	1.7	1.3	2.3	-43
2	16	-15	3.2	0.8	0.6	2.2	1.2	2.3	-47
2	15	-15	2.7	1.6	0.8	1.6	1.3	2.3	-42
3	incident	0	4.9	3.0	2.8	2.9	2.9	3.0	-3
3	25	0	3.8	3.2	3.5	3.4	3.4	3.0	13
3	23	0	4.1	3.2	2.7	3.3	3.1	3.0	2
3	21	0	4.3	2.7	2.4	2.7	2.6	3.0	-13
3	19	0	4.3	2.3	0.4	2.3	1.6	3.0	-46
3	18	0	4.3	2.1	1.2	1.5	1.6	3.0	-46
3	17	0	4.5	1.9	1.3	1.5	1.6	3.0	-48
3	16	0	4.5	2.8	1.0	1.5	1.8	3.0	-41
3	15	0	4.5	2.3	1.2	0.7	1.4	3.0	-53
4	incident	-15	4.9	3.4	2.5	4.0	3.3	3.0	10
4	25	-15	5.7	2.9	2.8	3.7	3.1	3.0	4
4	23	-15	5.8	3.7	3.1	3.4	3.4	3.0	13
4	21	-15	5.8	3.0	3.2	1.7	2.6	3.0	-12
4	19	-15	6.2	2.3	0.0	1.3	1.2	3.0	-60
4	18	-15	6.2	0.2	0.7	1.9	0.9	3.0	-69
4	17	-15	6.1	2.3	1.3	2.6	2.1	3.0	-31
4	16	-15	5.7	1.2	2.9	1.2	1.8	3.0	-41
4	15	-15	5.8	1.9	1.0	0.8	1.2	3.0	-59
5	incident	0	3.6	2.2	2.2	2.3	2.3	2.3	-2
5	19	0	3.8	2.0	0.4	0.8	1.1	2.3	-54
5	17	0	3.9	1.9	0.8	1.0	1.2	2.3	-47
5	15	0	3.9	1.8	0.8	1.2	1.3	2.3	-44
6	incident	0	4.5	2.7	2.6	2.8	2.7	3.0	-10
6	19	0	4.7	2.4	0.6	1.3	1.4	3.0	-52
6	17	0	4.7	2.2	1.2	1.4	1.6	3.0	-46
6	15	0	4.7	2.2	1.1	1.5	1.6	3.0	-47

Table 6. Summary of wave field for Alternative 3A

Keystone_H_Alt3B									
Case	Y-Location	Theta [deg]	WB [cm]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean H1:H3	Target [cm]	Percent Diff [%]
1	incident	0	4.1	2.1	2.0	2.2	2.1	2.3	-8
1	25	0	4.0	2.4	2.1	2.3	2.3	2.3	-2
1	23	0	4.1	2.0	2.2	2.3	2.1	2.3	-7
1	21	0	4.5	2.3	2.4	2.2	2.3	2.3	-1
1	19	0	4.6	2.6	1.7	2.4	2.2	2.3	-5
1	18	0	4.4	2.1	1.0	1.8	1.6	2.3	-29
1	17	0	4.2	2.0	1.8	1.9	1.9	2.3	-18
1	16	0	4.4	2.0	1.9	2.4	2.1	2.3	-10
1	15	0	4.2	1.9	1.9	2.6	2.2	2.3	-6
2	incident	-15	3.0	3.3	2.2	1.6	2.4	2.3	4
2	25	-15	3.6	4.4	3.2	1.8	3.1	2.3	36
2	23	-15	3.5	3.1	3.4	2.5	3.0	2.3	28
2	21	-15	3.6	3.2	3.7	3.8	3.6	2.3	55
2	19	-15	3.5	0.6	3.2	2.8	2.2	2.3	-7
2	18	-15	3.7	1.8	1.7	3.1	2.2	2.3	-5
2	17	-15	3.6	0.7	2.8	3.9	2.5	2.3	7
2	16	-15	3.8	2.9	2.1	3.7	2.9	2.3	26
2	15	-15	3.4	3.0	0.8	2.9	2.2	2.3	-3
3	incident	0	4.7	2.8	2.8	3.2	2.9	3.0	-2
3	25	0	4.4	3.2	2.8	3.6	3.2	3.0	7
3	23	0	4.4	2.9	2.4	3.1	2.8	3.0	-6
3	21	0	4.8	2.7	2.9	2.4	2.7	3.0	-11
3	19	0	4.9	1.7	2.4	3.7	2.6	3.0	-14
3	18	0	5.0	2.1	2.3	2.3	2.2	3.0	-26
3	17	0	5.0	2.2	3.5	1.8	2.5	3.0	-16
3	16	0	5.0	2.1	3.1	1.7	2.3	3.0	-23
3	15	0	5.2	1.6	2.7	0.7	1.7	3.0	-43
4	incident	-15	4.3	3.2	2.5	3.6	3.1	3.0	3
4	25	-15	5.2	3.3	2.8	3.8	3.3	3.0	11
4	23	-15	5.4	3.0	3.1	3.1	3.1	3.0	2
4	21	-15	5.5	3.3	3.6	2.0	3.0	3.0	-1
4	19	-15	5.5	2.1	3.0	4.1	3.1	3.0	3
4	18	-15	5.6	1.3	1.9	4.6	2.6	3.0	-14
4	17	-15	5.3	2.9	1.8	5.3	3.3	3.0	11
4	16	-15	5.2	3.5	1.3	3.2	2.7	3.0	-10
4	15	-15	5.3	3.7	0.7	2.3	2.2	3.0	-26
5	incident	0	3.6	2.1	1.9	2.3	2.1	2.3	-9
5	19	0	3.9	1.9	1.6	2.2	1.9	2.3	-17
5	17	0	3.9	1.7	2.1	1.7	1.8	2.3	-20
5	15	0	3.9	1.5	2.2	1.5	1.7	2.3	-24
6	incident	0	4.5	2.9	2.7	2.9	2.8	3.0	-7
6	19	0	4.7	2.2	1.8	2.9	2.3	3.0	-24
6	17	0	4.7	2.1	3.0	1.9	2.3	3.0	-22
6	15	0	4.7	2.1	2.5	1.8	2.1	3.0	-29

Table 7. Summary of wave field for Alternative 3B

Keystone_H_Alt4									
Case	Y-Location	Theta [deg]	WB [cm]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean H1:H3	Target [cm]	Percent Diff [%]
1	incident	0	3.2	2.2	2.2	1.9	2.1	2.3	-7
1	25	0	2.6	1.6	2.1	1.9	1.9	2.3	-19
1	23	0	2.8	2.2	1.7	2.5	2.2	2.3	-6
1	21	0	3.1	2.2	2.1	2.0	2.1	2.3	-9
1	19	0	3.2	2.0	0.4	0.3	0.9	2.3	-60
1	18	0	3.3	2.5	0.3	0.9	1.2	2.3	-47
1	17	0	3.3	2.6	0.4	0.4	1.1	2.3	-51
1	16	0	3.3	2.5	0.3	1.6	1.5	2.3	-37
1	15	0	3.2	2.0	0.3	0.9	1.0	2.3	-55
2	incident	-15	2.6	3.4	2.5	1.3	2.4	2.3	5
2	25	-15	2.5	4.3	2.6	0.8	2.6	2.3	13
2	23	-15	2.9	4.6	4.2	3.5	4.1	2.3	80
2	21	-15	2.8	2.8	3.9	3.2	3.3	2.3	45
2	19	-15	2.9	0.9	1.4	0.5	1.0	2.3	-58
2	18	-15	2.8	2.4	1.3	1.0	1.6	2.3	-31
2	17	-15	2.6	1.4	0.6	1.0	1.0	2.3	-56
2	16	-15	2.6	2.7	1.5	0.9	1.7	2.3	-26
2	15	-15	2.6	1.2	0.8	0.7	0.9	2.3	-60
3	incident	0	4.2	3.1	3.0	2.8	3.0	3.0	-1
3	25	0	4.0	3.1	2.7	3.1	3.0	3.0	-1
3	23	0	4.3	3.0	2.4	3.3	2.9	3.0	-3
3	21	0	4.6	3.2	3.6	3.7	3.5	3.0	18
3	19	0	4.7	3.0	1.0	1.5	1.8	3.0	-39
3	18	0	4.8	2.8	1.0	1.8	1.9	3.0	-37
3	17	0	4.6	2.6	1.7	0.6	1.6	3.0	-46
3	16	0	4.7	2.2	1.1	2.1	1.8	3.0	-40
3	15	0	4.7	1.9	1.2	2.1	1.7	3.0	-42
4	incident	-15	3.6	3.6	2.7	3.0	3.1	3.0	2
4	25	-15	4.2	4.1	3.0	3.3	3.5	3.0	14
4	23	-15	4.9	4.2	3.7	4.0	3.9	3.0	31
4	21	-15	4.9	3.3	2.7	2.6	2.9	3.0	-4
4	19	-15	5.1	3.6	0.3	1.0	1.6	3.0	-45
4	18	-15	5.0	0.9	0.8	1.2	1.0	3.0	-67
4	17	-15	4.5	2.3	1.1	1.5	1.6	3.0	-45
4	16	-15	4.6	1.5	2.2	1.0	1.6	3.0	-48
4	15	-15	4.2	1.9	1.1	0.5	1.2	3.0	-61
5	incident	0	3.0	2.3	2.0	2.0	2.1	2.3	-8
5	19	0	3.0	2.4	0.6	0.9	1.3	2.3	-44
5	17	0	3.1	1.8	0.7	1.1	1.2	2.3	-47
5	15	0	3.1	1.9	0.9	1.1	1.3	2.3	-44
6	incident	0	3.7	3.1	2.9	2.8	2.9	3.0	-2
6	19	0	3.8	2.9	0.9	1.3	1.7	3.0	-43
6	17	0	3.8	2.6	1.2	1.5	1.8	3.0	-41
6	15	0	3.8	2.3	1.2	1.5	1.7	3.0	-43

Table 7. Summary of wave field for Alternative 4A

Reference:

Lilly, K. Jr., **CH2M HILL,** 2002. Ferry Terminal Feasibility Study: Winds, Waves, and Currents. pp. 1-17

Hughes, S.A. 1993. Physical Models and Laboratory Techniques in Coastal Engineering, World Scientific, Singapore, pp.568.

Hughes, S.A., and Schwichtenberg, B.R. 1998. Current-induced scour along a breakwater at Ventura Harbor, CA – experimental study. Coastal Engineering, Vol. 34 (1998), pp 1-22.

Seabergh, W.C. and Smith, J.M. 2002. Wave Scaling in Tidal Inlet Physical Models. Ocean Wave Measurements and Analysis. pp. 1228-1237.

Simons, R.R., Whitehouse, R. JS., MacIver, R.D., Pearson, J., Sayers, P.B., Zhao, Y., and Channell, A.R. 1995. Evaluation of the UK Coastal Research Facility. Proc. Of Coastal Dynamics '95, pp 161-172, Gdansk, Poland.